GEOTECHNICAL REPORT

SOLID WASTE TRANSFER STATION
BRENTWOOD, CALIFORNIA

Submitted to:

Camp Dresser & McKee, Inc.
Mr. Wayne Pickus
100 Pringle Avenue, Suite 300
Walnut Creek, California 94596

July 28, 2009
Revised September 21, 2009
Project No. 8708.000.000

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INCORPORATED

GEOTECHNICAL
ENVIRONMENTAL
WATER RESOURCES
CONSTRUCTION SERVICES

Project No.
8708.000.000

July 28, 2009
Revised September 21, 2009

Mr. Wayne Pickus
Camp Dresser & McKee, Inc.
100 Pringle Avenue, Suite 300
Walnut Creek, California 94596

Subject: Solid Waste Transfer Station
Brentwood, California

GEOTECHNICAL REPORT

Dear Mr. Pickus:

ENGEO Incorporated prepared this geotechnical report for the proposed Solid Waste Transfer Station as outlined in our agreement dated May 6, 2009. We characterized the subsurface conditions at the proposed building site to provide the enclosed geotechnical recommendations for design.

Our experience, and that of our profession, clearly indicates that the risk of costly design, construction, and maintenance problems can be significantly lowered by retaining the design geotechnical engineering firm to review the project plans and specifications and provide geotechnical observation and testing services during construction. Please let us know when working drawings are nearing completion, and we will be glad to discuss these additional services with you.

If you have any questions or comments regarding this report, please call and we will be glad to discuss them with you.

Very truly yours,

ENGEO Incorporated

Steve Harris, GE

Josef J. Tootle, GE

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sh/jjt/dt

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1.0 INTRODUCTION

ENGEO Incorporated prepared this geotechnical report for design of the proposed Solid Waste Transfer Station in Brentwood, California. This report contains geotechnical recommendations for design of a new transfer station structure and access road to be located north of Elkins Way, adjacent to the wastewater treatment facility in the City of Brentwood.

1.1 SCOPE OF SERVICES

ENGEO prepared this report as outlined in our agreement dated May 6, 2009. Camp Dresser & McKee, Inc. (CDM) authorized ENGEO to conduct the proposed scope of services, which included the following:

- Exploratory drilling and sampling.
- Soil laboratory testing.
- Analysis of the geological and geotechnical data.
- Preparation of this report summarizing our findings and recommendations for site development.

1.2 PROJECT LOCATION

The proposed Solid Waste Transfer Station site is located north of Elkins Way, adjacent to the wastewater treatment facility in Brentwood, California, as shown on the Vicinity Map, Figure 1. The site is bound by dry ponds and Marsh Creek trail to the north and west, a former pond and agricultural property to the east and the Brentwood Wastewater Treatment Facility to the south.

Figure 2 shows the approximate locations of our exploratory borings and Cone Penetration Test (CPT). The project site consisted of a dry pond formerly associated with the wastewater treatment facility. The surface of the site consisted mainly of disked native soil and moderate to sparse coverage of native vegetation. In general, the topsoil consisted of eolian sand relatively free of organics. The bottom of the existing pond had been recently disked and was at an elevation of approximately 48 feet above mean sea level (msl). The bottom of the pond was approximately 10 feet below the surrounding property and had relatively gentle side slopes.

The project site also included a potential borrow area located northwest of the proposed Transfer Station site and a proposed access road to be located east and southeast of the site providing access from the east end of Elkins Way. The borrow area consisted of approximately 4 to 6 feet of undocumented fill most likely derived from the pond excavations.

1.3 PROJECT DESCRIPTION

Based on our discussions with the City of Brentwood and CDM and review of the information provided, we understand that site improvements will include:

1. Earthwork consisting primarily of cutting from the borrow site and filling or partially filling the dry pond to achieve design grades.

2. Construction of an approximately 28,000 square-foot transfer station and associated parking lot.

3. Access roads including fire lanes and underground transfer truck loading.

4. Underground utilities and other infrastructure improvements.

2.0 FINDINGS

We visited the site on May 15, 2009, to perform our site exploration. Section 2 presents descriptions of surface and subsurface conditions observed during our exploration.

2.1 SEISMIC SETTING

The site is located in an area of moderate seismicity. No known active[1] faults cross the property and the site is not located within an Alquist-Priolo Earthquake Fault Zone; however, large (>M_w7) earthquakes have historically occurred in the Bay Area and along the margins of the Central Valley and many earthquakes of low magnitude occur every year. The two nearest earthquake faults zoned as active by the State of California Geological Survey are the Greenville fault, located about 10 miles to the southwest, and the Great Valley fault, also located about 10 miles to the west (Blake, 2000). The Great Valley fault is a blind thrust fault with no known surface expression; the postulated fault location has been based on regional seismic activity and isolated subsurface information.

Portions of the Great Valley fault are considered seismically active thrust faults; however, since the Great Valley fault segments are not known to extend to the ground surface, the State of California has not defined Earthquake Fault Hazard Zones around the postulated traces. The Great Valley fault is considered capable of causing significant ground shaking at the site, but the recurrence interval is believed longer than for more distant, strike-slip faults. Recent studies suggest that this boundary fault may have been the cause of the Vacaville-Winters earthquake sequence of April 1892 (Eaton, 1986; Wong and Biggar, 1989; Moores and others, 1991). Further seismic activity can be expected to continue along the western margin of the Central Valley, and as with all projects in the area, the development should be designed to accommodate strong earthquake ground shaking.

[1] An active fault is defined by the State Mining and Geology Board as one that has had surface displacement within Holocene time (about the last 11,000 years). The State of California has prepared maps designating zones for special studies that contain these active earthquake faults.

Other active faults in the San Francisco Bay Area capable of producing significant ground shaking at the site include: the Mount Diablo thrust fault, 15 miles west; the Concord-Green Valley fault, 17 miles west; the Calaveras fault, 20 miles southwest; the Hayward fault, 29 miles southwest; and the San Andreas fault, 46 miles southwest. Any one of these faults could generate an earthquake capable of causing strong ground shaking at the subject site. Earthquakes of Moment Magnitude (Mw) 7 and larger have historically occurred in the Bay Area and numerous small magnitude earthquakes occur every year.

2.2 SITE GEOLOGY

We present the following discussion of site geology based on our field reconnaissance, subsurface exploration, and review of the CGS *Geologic Map of the San Francisco-San Jose Quadrangle* (Wagner, Bortugno, and McJunkin 1991).

The site is located in the Great Valley geomorphic province. The Great Valley is an elongate, northwest trending structural trough bound by the Coast Range on the west and the Sierra Nevada on the east. The Great Valley has been, and is presently being, filled with sediments primarily derived from the Sierra Nevada.

Our site reconnaissance and previously referenced geologic map indicate that near surface geology consists of Quaternary Dune Sand (Qs) underlain by Holocene-aged alluvial fan deposits (Qf) generally consisting of interbedded clay, silt, sand, and gravel.

2.3 SUBSURFACE CONDITIONS

On May 15, 2009, we observed drilling of four borings ranging in depths of approximately 16½ to 25 feet and performed four hand auger borings to depths of approximately 5 feet. In addition, we performed one CPT to a depth of approximately 70 feet. The locations of our explorations are shown on the Site Plan, Figure 2.

The soils encountered within the pond (Explorations 1-B1 through 1-B4 and 1-CPT1) generally consisted of very loose to medium dense sand, to a depth of approximately 20 feet, underlain by medium stiff to stiff silty clay, to a depth of approximately 32 feet. Beneath the silty clay layer, we encountered dense to very dense sand and silty sand with thin clay interbeds to a depth of approximately 43 feet. This sand layer was underlain by stiff to very stiff clay and silty clay to the maximum dept explored of 70 feet. The soils encountered at the potential borrow site consisted of sand and silty sand to a depth of 5 feet in Boring 1-B5 and silty clay and silty sand to a depth of 5 feet in Boring 1-B6. The borrow site materials were inconsistent and may be difficult to segregate. One plasticity index (PI) test was performed on the clayey material encountered at the borrow site and resulted in a PI of 25. This is an indication that some of the borrow site soils have a medium to high shrink-swell potential and should be considered moderately to highly expansive when subjected to fluctuations in moisture content. However, based on our observations, it appeared that sand was the predominate material and when blended with the clay, would likely have a low plasticity.

We also performed two hand auger borings (1-B7 and 1-B8) along the alignment of the proposed access road. The soils encountered in these explorations consisted of silty clay and clayey sand to a depth of 5 feet in Boring 1-B7 and sandy silt, clayey sand, and silty sand to a depth of 5 feet in Boring 1-B8. One bulk R-Value sample was collected off the near-surface soil at the location of Boring 1-B8 and resulted in an R-Value of 30.

We did not encounter any noticeably weak or compressible native soil in our exploratory borings.

Consult the Site Plan and boring logs for specific soil and groundwater conditions at the boring locations. Our boring logs are included in Appendix A. The logs contain the soil type, color, consistency, and visual classification in general accordance with the Unified Soil Classification System. Appendix A also provides additional exploratory information in the general notes to the logs.

2.4 GROUNDWATER CONDITIONS

Groundwater was measured in exploratory borings 1-B1 through 1-B4 at depths ranging from 11 to 14 feet below the ground surface. Groundwater at the site ranged in elevation from 34 to 38 feet above mean sea level (msl). Due to Contra Costa County geotechnical boring permit requirements, borings could not be left open for sufficient periods of time to establish equilibrium groundwater conditions; therefore, stabilized depths to groundwater could not generally be measured accurately. The depth of the groundwater encountered in this study should be used for preliminary design purposes. In addition, the groundwater elevation may fluctuate due to seasonal variation in rainfall, irrigation, or other factors not in evidence at the time of our exploration. To most accurately measure a "design" groundwater level, a groundwater piezometer should be installed at the site; however, a conservative high groundwater of elevation 40 feet may be used for design.

2.5 LABORATORY TESTING

We performed laboratory tests on selected soil samples to determine their engineering properties. For this project, we performed Plasticity Index testing, R-Value testing, moisture content, dry density, and sieve analysis. Selected laboratory test results are on the boring logs while individual test reports are in Appendix B.

2.6 CORROSION TESTING

To provide a preliminary corrosion evaluation, one near-surface sample was collected from Boring B1, at approximately 1.5 feet below the surface, submitted to Sunland Analytical and analyzed for pH, Minimum Resistivity, Sulfate, and Chloride. A copy of the laboratory report is presented in Appendix B. The laboratory results were compared to the 2007 CBC and the corrosion guidelines prepared by California Department of Transportation, Division of Engineering Services, Materials Engineering and Testing Services Corrosion Technology Branch; Version 1.0; September 2003. The information below is for informational purposes, for specific or long-term corrosion control design, we recommend contacting a corrosion specialist.

The California Building Code (CBC, 2007 Edition) Section 1904.3 references American Concrete Institute (ACI) 318, Section 4.3 which provides the following guidelines for cement type, maximum water cement ratio, and compressive strength for various sulfate concentrations, as summarized below:

Sulfate Exposure	Sulfate In Soil		Cement Type	Maximum Water-Cement Ratio	Minimum F'$_c$ (psi)
	Mg/kg	(%)			
Negligible	0 – 1,000	0.00 – 0.10	---	---	---
Moderate	1,000 – 2,000	0.10 – 0.20	II, IP(MS), IS(MS)	0.50	4,000
Severe	2,000 – 20,000	0.20 – 2.00	V	0.45	4,500
Very Severe	Over 20,000	over 2.00	V plus pozzolan	0.45	4,500

A sulfate content of 267.6 ppm (Mg/kg) was detected in the sample collected. As shown in the above table, this falls in the negligible sulfate exposure range.

The analysis of the soil resulted in a chloride content of 14.5 ppm detected in the sample. Caltrans Corrosion Guidelines (September 2003) considers a site to be corrosive if chloride concentrations are 500 ppm or greater. Therefore, the chloride content in the sample collected is not considered corrosive by these standards.

The minimum soil resistivity within the soil sample was 2,280 ohm-cm. Caltrans Corrosion Guidelines (September 2003) indicates that a minimum resistivity value of soil and water of less than 1,000 ohm-cm indicates the presence of high quantities of soluble salts and a higher propensity for corrosion. Based on these guidelines, the minimum soil resistivity in the sample collected is not considered corrosive by these standards. For specific or long-term corrosion control design, we recommend contacting a corrosion specialist.

The analysis of the soil resulted in a pH 4.41. Caltrans Corrosion Guidelines (September 2003) considers a site to be corrosive if the pH is 5.5 or less. Therefore, the pH in the sample collected **is considered corrosive** by these standards. Per the Caltrans Corrosion Guidelines, the on-site soil can react with the lime to form soluble reaction products that can easily leach out of the concrete. The result is a more porous, weaker concrete. For specific or long-term corrosion control design, we recommend contacting a corrosion specialist.

3.0 CONCLUSIONS AND RECOMMENDATIONS

Based on our exploration and laboratory test results, we conclude that the proposed project is feasible from a geotechnical and geologic standpoint. The recommendations included in this report, along with other sound engineering practices, should be incorporated in the design and construction of the project. The site was evaluated with respect to known geological and geotechnical hazards common to the region. The primary hazards identified are described below. None of the hazards listed below are considered unique to the property but are common to many sites in the region.

3.1 SEISMIC HAZARDS

Potential seismic hazards resulting from a nearby moderate to major earthquake including the Maximum Credible Earthquake (MCE) can generally be classified as primary and secondary. The primary effect is ground rupture, also called surface faulting. The common secondary seismic hazards include ground shaking, soil liquefaction, densification, lateral spreading, and flooding. These hazards are discussed in the following sections. Based on topographic and lithologic data, the risk of tsunamis or seiches is considered negligible at the site.

3.1.1 Ground Rupture

Since there are no known active faults crossing the property, and the site is not located within an Alquist-Priolo Earthquake Fault Zone, it is our opinion that primary fault ground rupture is unlikely at the property.

3.1.2 Ground Shaking

A potential seismic hazard at the site is strong ground shaking from a nearby moderate to major seismic event such as the MCE. The degree of shaking experienced at a site is dependent on the magnitude of the event, the distance to its epicenter, and the nature of the underlying soils. Based on the USGS Seismic Hazard Curves and Uniform Hazard Response Spectra 2002 Data Edition, a horizontal ground surface acceleration of 0.32g is predicted to have a 10 percent probability of being exceeded in a 50-year design life at the site. Additionally, a horizontal ground surface acceleration of 0.54g is predicted to have a 2 percent probability of being exceeded in a 50-year design life at the site.

To mitigate the ground shaking effects, all structures should be designed using sound engineering judgment and the latest California Building Code (CBC) requirements as a minimum. 2007 CBC Seismic Design Parameters are provided below in Section 3.2.

3.1.3 Liquefaction

Liquefaction is a phenomenon in which saturated cohesionless soils (sands and low plasticity silts) are subject to a temporary, but essentially total, loss of shear strength because of pore pressure build-up under the reversing cyclic shear stresses associated with earthquakes. As a result, these soils are temporarily transformed into a near liquid state. This process occurs most commonly in loose sands associated with a high groundwater table. One consequence that may result from the occurrence of liquefaction is an associated surface expression. If the seismic event occurs over an extended duration, the liquefied soils may migrate toward the surface, resulting in ejection and subsequent sand boiling at the surface. If not mitigated, this phenomenon of surface expression can result in ground settlement and disruption.

We evaluated the liquefaction potential at the site by measuring penetration resistance using the Cone Penetrometer Tests (CPT). A liquefaction analysis was conducted for the Cone Penetration Test and four exploratory borings located in the area of the proposed transfer station structure. The analyses generally followed guidelines provided by Robertson and Wride (1997). Based on this evaluation, it appears that the area beneath the proposed transfer station is underlain by potentially liquefiable material that is approximately 7 to 8 feet thick and is located within the upper 20 feet of the site.

3.1.3.1 Liquefaction-Induced Surface Rupture

In addition to the above analysis, we also evaluated the capping effect of any overlying non-liquefiable soils. In order for liquefaction-induced ground failure to occur, the pore water pressure generated within the liquefied strata must exert a sufficient enough force to break through the overlying soil and vent to the surface resulting in sand boils or fissures.

In 1985, Ishihara presented preliminary empirical criteria to assess the potential for ground surface disruption at liquefiable sites based on the relationship between thickness of liquefiable sediments and thickness of overlying non-liquefiable soil. A more recent study by Youd and Garris (1995) expanded on the work of Ishihara to include data from over 308 exploratory borings, 15 different earthquakes, and several ranges of recorded peak ground acceleration.

Based on the above studies, the potentially liquefiable soils at the site are currently capped by a sufficient thickness of non-liquefiable soils to prevent venting and surface rupture or sand boils during a strong seismic event.

3.1.3.2 Liquefaction-Induced Settlement

Densification of the sandy soils below groundwater levels can result in associated settlements during a design level earthquake or MCE. Based on the methods by Tokimatsu and Seed (1987), we estimate that volumetric strains of approximately 2 percent could be expected during a design-basis earthquake (an earthquake event producing a site acceleration with a 10 percent probability of being exceeded in 50 years). We estimate the total liquefaction-induced settlements beneath the proposed structure could be as much as 2 inches. Differential settlement at this location during a liquefaction event is expected to be on the order of ½-inch.

3.1.4 Seismic-induced Settlement

Densification of loose granular soils above the groundwater level can cause settlement due to earthquake-induced vibrations. Due to the density of the granular materials sampled in the boring, the potential for significant densification of granular layers above the groundwater table due to earthquake shaking is considered low at the site.

3.1.5 Lateral Spreading

Lateral spreading is a failure within a nearly horizontal soil zone (possibly due to liquefaction) that causes the overlying soil mass to move toward a free face or down a gentle slope. Due to the distance between the project site and Marsh Creek, the effects of lateral spreading on the site are negligible.

3.2 2007 CBC SEISMIC DESIGN PARAMETERS

To provide California Building Code (CBC) seismic design parameters, we reviewed the 2007 CBC and the February 1998 California Divisions of Mines and Geology "Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada".

Based on our review, we provide the 2007 California Building Code (CBC) seismic parameters in Table 1 below.

Table 1
ASCE 7.05 / 2006 IBC/ 2007 CBC Seismic Design Parameters

Parameter	Design Value
Site Class	D
0.2 second Spectral Response Acceleration, S_S	1.31
1.0 second Spectral Response Acceleration, S_1	0.45
Site Coefficient, F_A	1.00
Site Coefficient, F_V	1.55
Maximum considered earthquake spectral response accelerations for short periods, S_{MS}	1.31
Maximum considered earthquake spectral response accelerations for 1-second periods, S_{M1}	0.70
Design spectral response acceleration at short periods, S_{DS}	0.87
Design spectral response acceleration at 1-second periods, S_{D1}	0.46
Long period transition-period, T_L	8 Seconds

3.3 EXPANSIVE SOILS

As discussed in previous sections of this report, the soil in the location of the proposed structure are sandy and generally nonexpansive. Some of the soil observed within the borrow area soils have a moderate to high expansion potential. Expansive soils shrink and swell as a result of moisture changes. This can cause heaving and cracking of slabs-on-grade, pavements and structures founded on shallow foundations. The clayey material located within the proposed borrow area adjacent to boring B-6 should not be placed within the footprint of the proposed structure. If it is deemed necessary to use expansive material beneath the proposed structure, supplemental geotechnical recommendations will be necessary.

4.0 EARTHWORK RECOMMENDATIONS

The relative compaction and optimum moisture content of soil and aggregate base referred to in this report are based on the most recent ASTM D1557 test method. Compacted soil is not acceptable if it is unstable. It should exhibit only minimal *flexing* or *pumping,* as determined by the geotechnical engineer.

As used in this report, the term "moisture condition" refers to adjusting the moisture content of the soil by either drying if too wet or adding water if too dry. We define "structural areas" in Section 4 of this report as any area sensitive to settlement of compacted soil. These areas include, but are not limited to, structure building pads, sidewalks, pavement areas, and retaining walls.

4.1 GENERAL SITE CLEARING

Clear areas to be developed of all surface and subsurface deleterious materials (if any is observed) including debris, shrubs, trees and associated root systems, and fencing. Clean and backfill excavations extending below the planned finished site grades with suitable material compacted to the recommendations presented in Section 4.4.

Following clearing, strip the site to remove surface organic materials. Strip organics from the ground surface to a depth of at least 2 to 3 inches below the surface. Remove strippings from the site or use them in landscape fill. It may also be feasible to mulch organics in place, depending on the amount and type of vegetation present at the time of grading as well as the proposed mulching method.

4.2 OVER-OPTIMUM SOIL MOISTURE CONDITIONS

The contractor should anticipate encountering excessively over-optimum (wet) soil moisture conditions during winter or spring grading, or during or following periods of rain. Wet soil can make proper compaction difficult or impossible. Wet soil conditions can be mitigated by:

1. Frequent spreading and mixing during warm dry weather;
2. Mixing with drier materials;
3. Mixing with a lime, lime-flyash, or cement product; or
4. Stabilizing with aggregate, geotextile stabilization fabric, or both.

Options 3 and 4 should be evaluated and approved by the geotechnical engineer prior to implementation.

4.3 LIQUEFACTION MITIGATION

As previously discussed, loose potentially liquefiable sands were encountered within the upper 20 feet of soil at the site. If the foundation cannot be designed to accommodate the anticipated settlement, ground improvements should be made to the proposed building pads including: over excavating to a sufficient depth to remove the potentially liquefiable soils or treating with Deep Dynamic Compaction (DDC), or other densification technology, to increase the density of the potentially liquefiable soils. The selected ground improvement method should be preformed a minimum of 5 feet beyond the proposed building footprint and primary utility corridors. If ground improvement is desired, ENGEO should be retained to observe ground improvement measures and perform confirmation testing to verify effectiveness. If the potential liquefaction-induced differential settlement discussed in section 3.1.3.2 is more than the proposed structure can tolerate, a geotextile reinforced fill pad can be constructed that can reduce the effects of the potential differential settlement on the structure. Supplemental recommendations for a geotextile reinforced fill pad can be provided by ENGEO if required. In addition, the associated underground utilities may require flexible connections and adequate fall to accommodate the expected settlement.

4.4 ACCEPTABLE FILL

On-site soil material is suitable as fill material provided it is processed to remove concentrations of organic material, debris, and particles greater than 6 inches in maximum dimension. The proposed borrow areas adjacent to boring B-5 and B-6 are generally acceptable with the exception of the surficial clayey material encountered in Boring B-6, which may require blending to produce a suitable fill material with a PI less than 10.

Imported fill materials should meet the above requirements and be consistent with the material properties of the on-site soil. Allow the geotechnical engineer to sample and test proposed imported fill materials at least 72 hours prior to delivery to the site.

4.5 FILL COMPACTION

The exposed, non-yielding surface to be filled over should be scarified to a minimum depth of 12 inches, moisture conditioned, and recompacted as engineered fill to provide adequate bonding with the initial lift of fill. All fills should be placed in uncompacted lifts not exceeding 12 inches or the depth of penetration of the compaction equipment used, whichever is less. In cut portions of the site, a 12-inch scarification, moisture conditioning, and recompaction of the exposed subgrade will be necessary, below the finished subgrade elevation.

The following compaction control recommendations should be applied to all engineered fills.

Test Procedures:	ASTM D-1557 (latest edition).
Required Moisture Content:	Above optimum moisture content.
Relative Compaction:	At least 95 percent relative compaction.

Compact the pavement Caltrans Class 2 Aggregate Base section, and other non-expansive import material, to at least 95 percent relative compaction (ASTM D1557). Moisture condition aggregate base to at least optimum moisture content prior to compaction.

4.6 GRADED SLOPES

Cut or fill slopes should be graded no steeper than 3:1 (horizontal:vertical). Structures located adjacent to slopes should be set-back from the top-of-slope a minimum of one-third the slope height or a minimum of 10 feet, whichever is greater.

4.6.1 Underground Utility Backfill

4.6.1.1 General

The contractor is responsible for conducting all trenching and shoring in accordance with CALOSHA requirements. Project consultants involved in utility design should specify pipe bedding materials. Trench backfill should be placed in accordance with the City of Brentwood specifications.

Jetting of backfill is not an acceptable means of compaction.

4.7 SITE DRAINAGE

4.7.1 Surface Drainage

The project civil engineer is responsible for designing surface drainage improvements. With regard to geotechnical engineering issues, we provide the following minimum recommendation for surface drainage.

- Slope pavement areas a minimum of 1 percent towards drop inlets or other surface drainage devices.

5.0 FOUNDATION RECOMMENDATIONS

5.1 FOUNDATION DESIGN

Provided that the site is graded and the building pad is prepared in accordance with the recommendations provided herein, the proposed building should be founded on a conventional perimeter strip and isolated interior footing system. The following sections provide recommendations for the proposed building foundation. Final foundation plans should be submitted for review to the geotechnical engineer prior to submittal to the appropriate agency.

5.2 CONVENTIONAL FOOTING SYSTEM

Conventional footings should be designed according to the following design criteria; these recommendations should be confirmed following mass grading:

Maximum Allowable Bearing Pressure:	3,000 psf for dead-plus-live loads. This value can be increased by 30 percent to include transient loads such as seismic or wind loads.
Minimum Depth of Footing:	18 inches below lowest pad grade.
Minimum Footing Width:	15 inches.

If liquefaction mitigation is not performed, the structural engineer should design the Transfer Station foundation so that it will be capable of accommodating a maximum liquefaction-induced differential settlement of ½-inch across the minimum building dimension without resulting in distress to the foundation or critical finishes of the structure.

The geotechnical engineer should review foundation plans when they become available. Footing trenches should be cleared of all loose materials, and soils exposed in footing excavations should not be allowed to dry out. The geotechnical engineer or his/her field representative should observe the footing trenches prior to concrete placement.

5.3 FOUNDATION LATERAL RESISTANCE

Lateral loads may be resisted by friction along the base and by passive pressure along the sides of foundations. The passive pressure is based on an equivalent fluid pressure in pounds per cubic foot (pcf). We recommend the following allowable values for design:

Passive Lateral Pressure: 300 pcf

Coefficient of Friction: 0.30

The above allowable values include a factor of safety of 1.5. Increase the above values by one-third for the short-term effects of wind or seismic loading.

Passive lateral pressure should not be used for footings on or above slopes.

6.0 SLABS-ON-GRADE

6.1 EXTERIOR FLATWORK

Exterior flatwork includes items such as concrete sidewalks and walkways exposed to foot traffic only. Provide a minimum concrete flatwork thickness of 4 inches. Construct control and construction joints in accordance with current Portland Cement Association Guidelines.

6.2 INTERIOR CONCRETE FLOOR SLABS

Interior slab-on-grade foundation construction for the structures in combination with conventional spread footings should be designed by the structural engineer for the anticipated loading conditions. A minimum concrete thickness of 5 inches should be considered and the slab reinforcement should consist of steel bars. It is our experience that welded wire mesh reinforcement is not effective in controlling cracks. The structural engineer should provide final design thickness and final reinforcement, as necessary, for the intended structural loads. Prior to construction, the subgrade should be moisture conditioned. The finished subgrade should be smooth and unyielding.

6.2.1 Slab Moisture Vapor Reduction

When buildings are constructed with concrete slab-on-grade, water vapor from beneath the slab will migrate through the slab and into the building. This water vapor can be reduced but not stopped. Vapor transmission can negatively affect floor coverings and lead to increased moisture within a building. When water vapor migrating through the slab would be undesirable, we recommend the following to reduce, but not stop, water vapor transmission upward through the slab-on-grade.

1. Construct a moisture retarder system directly beneath the slab-on-grade that consists of the following:

    a) Vapor retarder membrane sealed at all seams and pipe penetrations and connected to all footings. Vapor retarders shall conform to Class A vapor retarder per ASTM E 1745-97 "Standard Specification for Plastic Water Vapor Retarders used in Contact with Soil or Granular Fill under Concrete Slabs". The vapor retarder should be **underlain by -**

        i) 4 inches of clean crushed rock. Crushed rock should have 100 percent passing the ¾-inch sieve and less than 5 percent passing the No. 4 Sieve.

2. Use a concrete water-cement ratio for slabs-on-grade of no more than 0.50.

3. Provide inspection and testing during concrete placement to check that the proper concrete and water cement ratio are used.

4. Moist cure slabs for a minimum of 3 days or use other equivalent curing specific by the structural engineer.

The structural engineer should be consulted as to the use of a layer of clean sand or pea gravel (less than 5 percent passing the U.S. Standard No. 200 Sieve) placed on top of the vapor retarder membrane to assist in concrete curing.

6.2.2 Subgrade Modulus for Structural Slab Design

Provided the site earthwork is conducted in accordance with the recommendations of this report, a subgrade modulus of 120 psi/in can be used for structural slab design.

7.0 RETAINING WALLS

7.1 LATERAL SOIL PRESSURES

Design proposed retaining walls to resist lateral earth pressures from adjoining natural materials and/or backfill and from any surcharge loads. Provided that adequate drainage is included as recommended below, design walls restrained from movement at the top to resist an equivalent fluid pressure of 60 pounds per cubic foot (pcf). In addition, design restrained walls to resist an additional uniform pressure equivalent to one-half of any surcharge loads applied at the surface.

Design unrestrained retaining walls with adequate drainage to resist an equivalent fluid pressure of 40 pcf plus one-third of any surcharge loads.

The above lateral earth pressures assume level backfill conditions and sufficient drainage behind the walls to prevent any build-up of hydrostatic pressures from surface water infiltration and/or a rise in the groundwater level. If adequate drainage is not provided, we recommend that an additional equivalent fluid pressure of 40 pcf be added to the values recommended above for both restrained and unrestrained walls. Damp-proofing of the walls should be included in areas where wall moisture would be problematic.

For seismic lateral loading conditions, we recommend and additional load of $22H^2$ pounds per foot of wall length, acting at a height of 0.6H above wall base. For this application, H is the height of retained soil.

Construct a drainage system, as recommended below, to reduce hydrostatic forces behind the retaining wall.

7.2 RETAINING WALL DRAINAGE

Construct either graded rock drains or geosynthetic drainage composites behind the retaining walls to reduce hydrostatic lateral forces. For rock drain construction, we recommend two types of rock drain alternatives:

1. A minimum 12-inch-thick layer of Class 2 Permeable Filter Material (Caltrans Specification 68-1.025) placed directly behind the wall, or

2. A minimum 12-inch-thick layer of washed, crushed rock with 100 percent passing the ¾-inch sieve and less than 5 percent passing the No. 4 sieve. Envelope rock in a nonwoven geotextile filter fabric such as Mirafi 140NC, or equivalent.

For both types of rock drains:

1. Place the rock drain directly behind the walls of the structure.

2. Extend rock drains from the wall base to within 12 inches of the top of the wall.

3. Place a minimum of 4-inch-diameter perforated pipe at the base of the wall, inside the rock drain and fabric, with perforations placed down.

4. Place pipe at a gradient at least 1 percent to direct water away from the wall by gravity to a drainage facility.

The geotechnical engineer should review the geosynthetic composite drainage system design prior to use.

7.3 BACKFILL

Backfill behind retaining walls should be placed and compacted in accordance with Section 4.4. Use light compaction equipment within 5 feet of the wall face. If heavy compaction equipment is used, the walls should be temporarily braced to avoid excessive wall movement.

7.4 FOUNDATIONS

Retaining walls may be supported on continuous footings designed in accordance with recommendations presented in Sections 5.2 and 5.3.

8.0 PRELIMINARY PAVEMENT DESIGN

8.1 FLEXIBLE PAVEMENTS

As previously discussed, one R-Value sample was collected for preliminary pavement design purposes. Based on our laboratory testing an R-value of 30 was used. Using estimated traffic indices for various pavement loading requirements, we developed the following recommended pavement sections using Procedure 608 of the Caltrans Highway Design Manual (including the asphalt factor of safety), presented in Table 2 below.

Table 2
Recommended Asphalt Concrete Pavement Sections – R-Value 30

	Section	
Traffic Index	Asphalt Concrete (in.)	Class 2 Aggregate Base (in.)
4.5	4.0*	8.0*
5	4.0*	8.0*
6	4.0*	8.0
7	4.0	10.0
8	5.0	11.0
9	5.5	13.0
10	6.5	14.0
11	7.0	16.0

Notes: * Minimum pavement section component thickness as required by City of Brentwood
AC is asphaltic concrete
AB is aggregate base Class 2 Material with minimum R = 78

If moderately expansive clayey material is encountered at subgrade, additional R-Value samples should be collected; however, a conservative design based on an assumed R-Value of 10 is presented below in Table 3

Table 3
Recommended Asphalt Concrete Pavement Sections – R-Value 10

	Section	
Traffic Index	Asphalt Concrete (in.)	Class 2 Aggregate Base (in.)
4.5	4.0*	8.0*
5	4.0*	8.0*
6	4.0*	11.0
7	4.0	15.0
8	5.0	16.0
9	5.5	19.0
10	6.5	21.0
11	7.0	24.0

Notes: * Minimum pavement section component thickness as required by City of Brentwood
AC is asphaltic concrete
AB is aggregate base Class 2 Material with minimum R = 78

The civil engineer should determine the appropriate traffic indices based on the estimated traffic loads and frequencies.

8.2 SUBGRADE AND AGGREGATE BASE COMPACTION

Subgrade soils should be in a stable, non-pumping condition at the time aggregate baserock materials are placed and compacted. Proof-rolling with a heavy wheel-loaded piece of construction equipment should be implemented. Yielding materials should be appropriately mitigated, with suitable mitigation measures developed in coordination with the client, contractor, and Geotechnical Engineer. Compact finish subgrade and aggregate base in accordance with Section 4.4. Aggregate Base should meet the requirements for ¾-inch maximum Class 2 AB per section 26-1.02a of the latest Caltrans Standard Specifications.

All concrete curbs separating pavement and irrigated landscaped areas should extend into the subgrade and below the bottom of adjacent aggregate baserock materials. An undercurb drain should also be considered to help collect and transport subsurface seepage.

9.0 LIMITATIONS

This report presents geotechnical recommendations for construction of improvements discussed in Section 1.3 for the proposed Solid Waste Transfer Station project. If changes occur in the nature or design of the project, we should be allowed to review this report and provide additional recommendations, if any.

We strived to perform our professional services in accordance with generally accepted geotechnical engineering principles and practices currently employed in the area; no warranty is expressed or implied.

We developed this report with limited subsurface exploration data. We assumed that our subsurface exploration data is representative of soil and groundwater conditions across the site. Considering possible underground variability of soil and groundwater, additional costs may be required to complete the project. We recommend that the owner establish a contingency fund to cover such costs. If unexpected conditions are encountered, notify ENGEO immediately to review these conditions and provide additional and/or modified recommendations, as necessary.

The location and elevations of our borings are approximate and were estimated by pacing from features shown on the Site Plan, Figure 2 prepared, May 2009.

Our services did not include excavation sloping or shoring, soil volume change factors, flood potential, or a geohazard exploration.

This geotechnical exploration did not include work to determine the existence of possible hazardous materials. If any hazardous materials are encountered during construction, then notify the proper regulatory officials immediately.

Our experience, and that of our profession, clearly indicates that the risk of costly design, construction, and maintenance problems can be significantly lowered by retaining the design geotechnical engineering firm to provide construction monitoring services. If we are not retained to perform the services described above, then we are not responsible for any party's interpretation of our report (and subsequent addenda, letters, and verbal discussions).

SELECTED REFERENCES

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FIGURES

Figure 1 – Vicinity Map
Figure 2 – Site Plan

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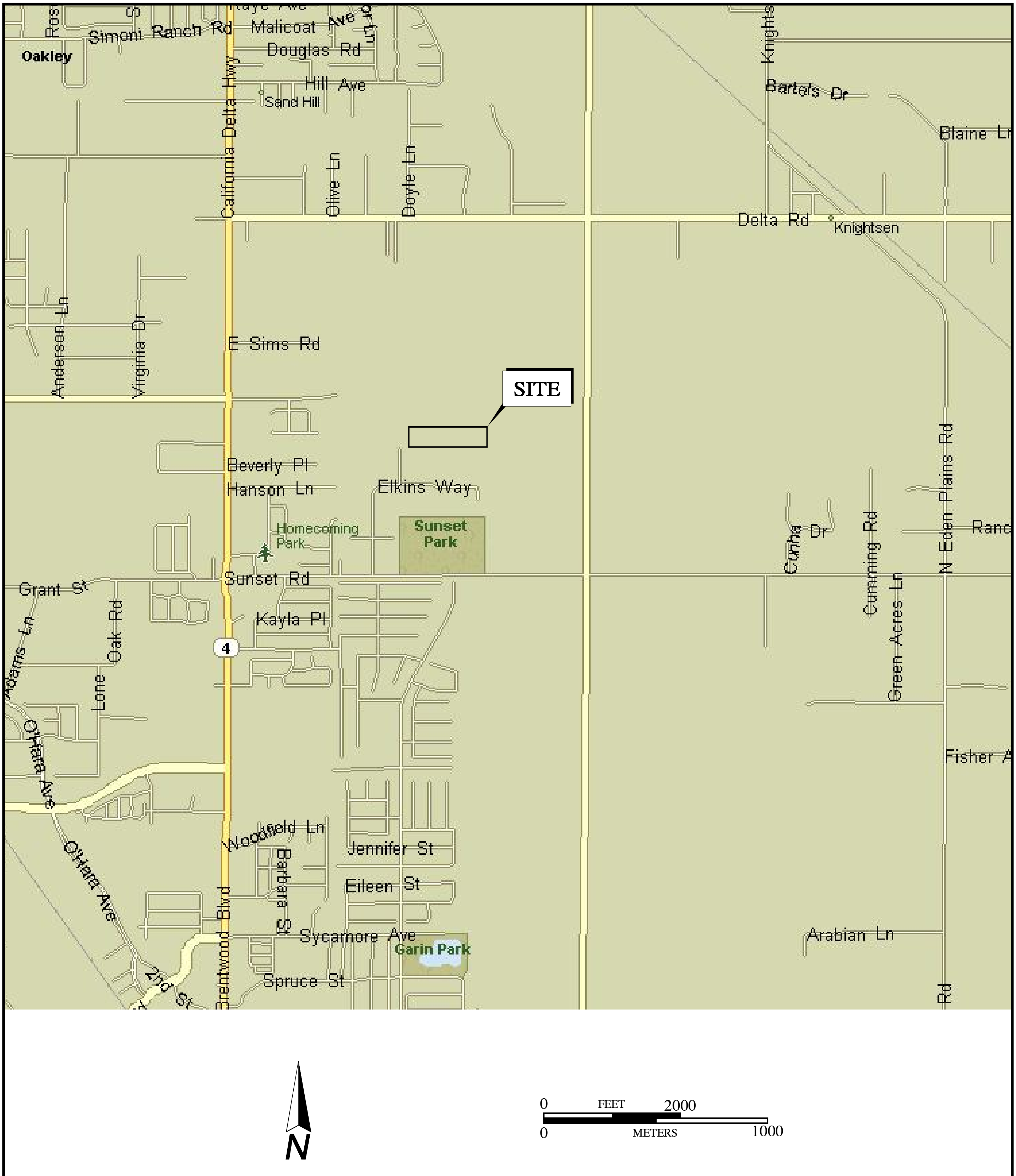


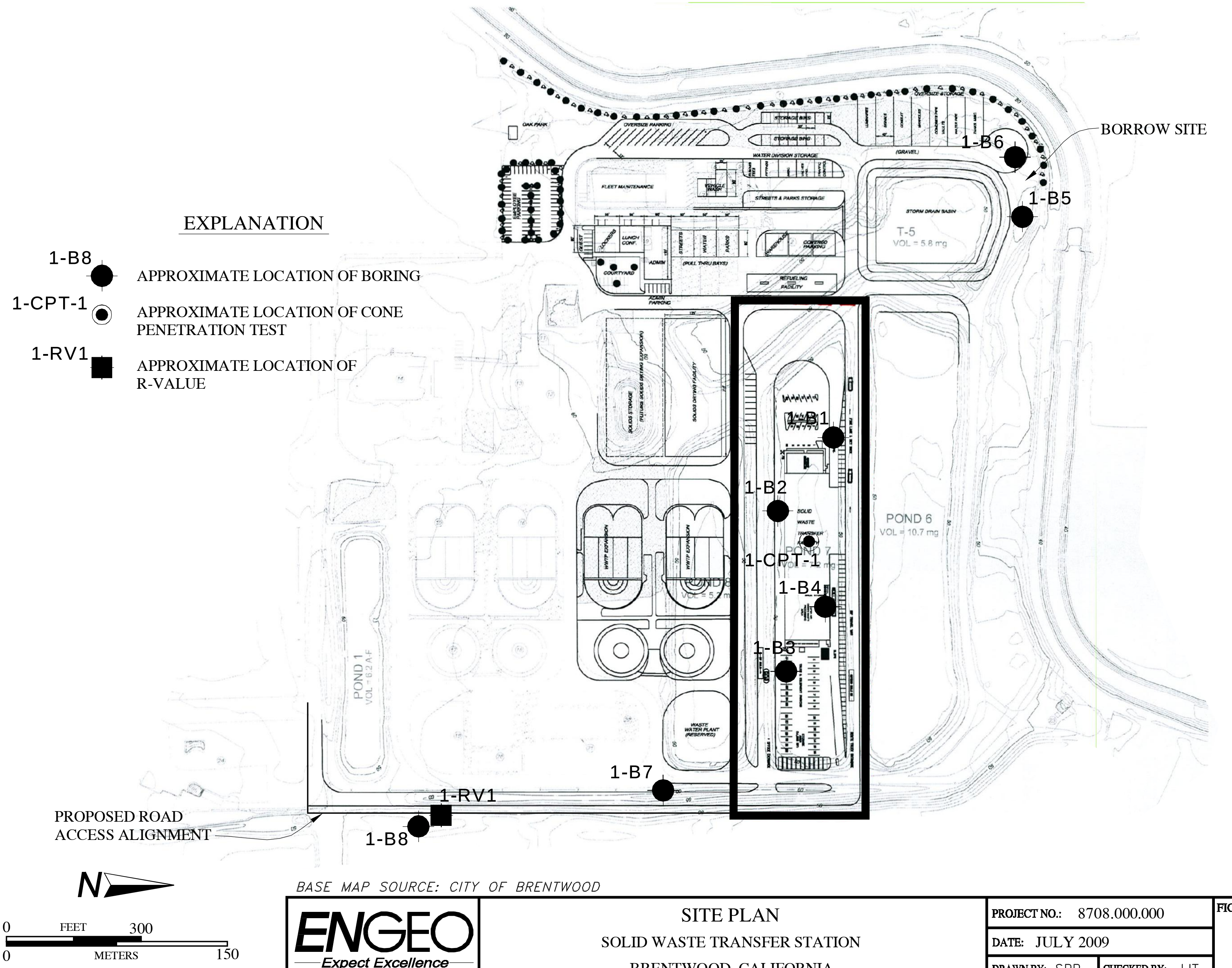
BASE MAP SOURCE: MS STREETS AND TRIPS

ENGEO — Expect Excellence —	VICINITY MAP SOLID WASTE TRANSFER STATION BRENTWOOD, CALIFORNIA	PROJECT NO.: 8708.000.000 DATE: JULY 2009 DRAWN BY: SRP CHECKED BY: JJT	FIGURE NO. 1

G:\Drafting\DRAFTING2_Dwg\8708\000\GEX\8708000000-1-VicinityMap-0709.dwg 7-29-09 07:40:46 AM spatters

ORIGINAL FIGURE PRINTED IN COLOR

G:\Drafting\DRAFTING2_Dwg\8708\000\GEX\8708000000-2-SitePlan-0709.dwg 7-29-09 07:41:52 AM spatters



BASE MAP SOURCE: CITY OF BRENTWOOD

ENGEO
Expect Excellence

SITE PLAN
SOLID WASTE TRANSFER STATION
BRENTWOOD, CALIFORNIA

PROJECT NO.: 8708.000.000	FIGURE NO. 2
DATE: JULY 2009	
DRAWN BY: SRP	CHECKED BY: JJT

APPENDIX A

Field Exploration Notes
Key to Boring Logs
Boring Logs
Cone Penetration Test Log

APPENDIX A

FIELD EXPLORATION NOTES

We drilled eight borings on the site for this report. An ENGEO representative supervised the drilling and logged the subsurface conditions. A Marl-11 drill rig equipped with 6-inch-diameter hollow stem augers was used to drill the borings 1-B1 through 1-B4. Borings 1-B5 through 1-B8 were drilled using standard hand auger equipment.

The boring logs present descriptions and graphically depict the subsurface soil conditions encountered. The maximum depth penetrated by the borings was 25 feet.

We retrieved both disturbed and relatively undisturbed soil samples at various intervals in the boring using standard penetration tests (SPT) and a Modified California Sampler (3-inch O.D. split spoon sampler with thin-walled liners).

The standard penetration resistance blow counts were obtained by dropping a 140-pound hammer through a 30-inch free fall employing a rope and cat-head system. The 2-inch O.D. split spoon sampler was driven 18 inches and the number of blows was recorded for each 6 inches of penetration. In addition, 2.5-inch I.D. samples were obtained using a Modified California Sampler driven into the soil with the 140-pound hammer previously described. Unless otherwise indicated, the blows per foot recorded on the boring logs represent the accumulated number of blows required to drive the last 12 inches. Disturbed samples obtained from the hand auger borings were collected by hand and placed in sealed bags for transport to our laboratory for testing.

NOTES TO THE LOGS

We determined the lines designating the interface between soil materials on the logs using visual observations. The transition between the materials may be abrupt or gradual.

The logs contains information concerning samples recovered, indications of the presence of various materials such as sand, silt, clay and observations of groundwater encountered. The field logs also contain our interpretation of the soil conditions between samples. Therefore, the logs contain both factual and interpretative information. Our recommendations are based on the contents of the final logs. The final logs represent our interpretation of the contents of the field logs.

KEY TO SOIL LOGS

MAJOR TYPES			DESCRIPTION
COARSE-GRAINED SOILS MORE THAN HALF OF MAT'L LARGER THAN #200 SIEVE	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE SIZE	CLEAN GRAVELS WITH LESS THAN 5% FINES	GW - Well graded gravels or gravel-sand mixtures
			GP - Poorly graded gravels or gravel-sand mixtures
		GRAVELS WITH OVER 12 % FINES	GM - Silty gravels, gravel-sand and silt mixtures
			GC - Clayey gravels, gravel-sand and clay mixtures
	SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE SIZE	CLEAN SANDS WITH LESS THAN 5% FINES	SW - Well graded sands, or gravelly sand mixtures
			SP - Poorly graded sands or gravelly sand mixtures
		SANDS WITH OVER 12 % FINES	SM - Silty sand, sand-silt mixtures
			SC - Clayey sand, sand-clay mixtures
FINE-GRAINED SOILS MORE THAN HALF OF MAT'L SMALLER THAN #200 SIEVE	SILTS AND CLAYS LIQUID LIMIT 50 % OR LESS		ML - Inorganic silt with low to medium plasticity
			CL - Inorganic clay with low to medium plasticity
			OL - Low plasticity organic silts and clays
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50 %		MH - Elastic silt with high plasticity
			CH - Fat clay with high plasticity
			OH - Highly plastic organic silts and clays
	HIGHLY ORGANIC SOILS		PT - Peat and other highly organic soils

For fine-grained soils with 15 to 29% retained on the #200 sieve, the words "with sand" or "with gravel" (whichever is predominant) are added to the group name.

For fine-grained soil with >30% retained on the #200 sieve, the words "sandy" or "gravelly" (whichever is predominant) are added to the group name.

GRAIN SIZES

U.S. STANDARD SERIES SIEVE SIZE — CLEAR SQUARE SIEVE OPENINGS

200		40	10	4		3/4 "	3"	12"
SILTS AND CLAYS	SAND			GRAVEL		COBBLES	BOULDERS	
	FINE	MEDIUM	COARSE	FINE	COARSE			

RELATIVE DENSITY

SANDS AND GRAVELS	BLOWS/FOOT (S.P.T.)
VERY LOOSE	0-4
LOOSE	4-10
MEDIUM DENSE	10-30
DENSE	30-50
VERY DENSE	OVER 50

CONSISTENCY

SILTS AND CLAYS	STRENGTH*
VERY SOFT	0-1/4
SOFT	1/4-1/2
MEDIUM STIFF	1/2-1
STIFF	1-2
VERY STIFF	2-4
HARD	OVER 4

SAMPLER SYMBOLS

- Modified California (3" O.D.) sampler
- California (2.5" O.D.) sampler
- S.P.T. - Split spoon sampler
- Shelby Tube
- Continuous Core
- Bag Samples
- Grab Samples
- NR No Recovery

MOISTURE CONDITION

DRY	Dusty, dry to touch
MOIST	Damp but no visible water
WET	Visible freewater

LINE TYPES

________	Solid - Layer Break
– – – – – –	Dashed - Gradational or approximate layer break

GROUND-WATER SYMBOLS

- ▽ Groundwater level during drilling
- ▼ Stabilized groundwater level

(S.P.T.) Number of blows of 140 lb. hammer falling 30" to drive a 2-inch O.D. (1-3/8 inch I.D.) sampler

* Unconfined compressive strength in tons/sq. ft., asterisk on log means determined by pocket penetrometer

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LOG OF BORING 1-B1

Geotechnical Exploration
Solid Waste Transfer Station
Brentwood, California
8708.000.000

DATE DRILLED: 5/15/2009
HOLE DEPTH: Approx. 25 ft.
HOLE DIAMETER: 6.0 in.
SURF ELEV (msl): Approx. 49 ft.

LOGGED / REVIEWED BY: Z. Crawford / JJT
DRILLING CONTRACTOR: Gregg Drilling & Testing
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: Automatic Trip Hammer

DESCRIPTION	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
SAND (SP), light yellowish brown, very loose to loose, dry to moist, fine-grained sand, <5% silt									
(moist, yellowish brown)		23				5	4.4		
		10							
(grades medium dense)									
		12					4.7		
(wet)	∇	15					13.8		
SILTY SAND (SM), grayish brown, medium dense, wet, fine-grained sand		11				24	20.3		
SAND (SP), grayish brown, medium dense, wet, fine-grained sand		11							
SILTY CLAY (CL), brown, stiff to very stiff, wet, <15% fine-grained sand									

Depth in Feet: 5, 10, 15, 20, 25
Depth in Meters: 1, 2, 3, 4, 5, 6, 7

Bottom of boring at approximately 25 feet.

Groundwater encountered at 11 feet during drilling.

LOG OF BORING 1-B2

ENGEO INCORPORATED

Geotechnical Exploration
Solid Waste Transfer Station
Brentwood, California
8708.000.000

DATE DRILLED: 5/15/2009
HOLE DEPTH: Approx. 16½ ft.
HOLE DIAMETER: 6.0 in.
SURF ELEV (msl): Approx. 50 ft.

LOGGED / REVIEWED BY: Z. Crawford / JJT
DRILLING CONTRACTOR: Gregg Drilling & Testing
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: Automatic Trip Hammer

Depth in Feet	Depth in Meters	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
			SAND (SP), light yellowish brown, very loose, dry, fine-grained sand, <10% silt										
			(moist, loose)										
	1					18							
5						11							
	2		(grades to medium dense)										
10	3					14							
	4		(wet)		▽								
15	5					13				6			
			Bottom of boring at approximately 16 1/2 feet.										
			Groundwater encountered at 14 feet during drilling.										

ENGEO INCORPORATED

LOG OF BORING 1-B3

Geotechnical Exploration
Solid Waste Transfer Station
Brentwood, California
8708.000.000

DATE DRILLED: 5/15/2009
HOLE DEPTH: Approx. 21½ ft.
HOLE DIAMETER: 6.0 in.
SURF ELEV (msl): Approx. 48 ft.

LOGGED / REVIEWED BY: Z. Crawford / JJT
DRILLING CONTRACTOR: Gregg Drilling & Testing
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: Automatic Trip Hammer

Depth in Feet	Depth in Meters	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
			SAND (SP), light yellowish brown, very loose, dry, fine-grained sand, <10% silt										
			(yellowish brown, loose to medium dense, moist)			17					4.6		
	1		(grades to loose)										
5	2					9					5.4		
10	3		(brown)			7					6.4		
	4		(wet, grades to <5% silt)		▽								
15	5					7				4	21		
20	6		SANDY CLAY (CL), yellowish brown and olive brown, stiff, wet, < 20% silt			9				64	22.8		

Bottom of boring at approximately 21 1/2 feet.

Groundwater encountered at 14 feet during drilling.

LOG OF BORING 1-B4

ENGEO INCORPORATED

Geotechnical Exploration
Solid Waste Transfer Station
Brentwood, California
8708.000.000

DATE DRILLED: 5/15/2009
HOLE DEPTH: Approx. 16½ ft.
HOLE DIAMETER: 6.0 in.
SURF ELEV (msl): Approx. 48 ft.

LOGGED / REVIEWED BY: Z. Crawford / JJT
DRILLING CONTRACTOR: Gregg Drilling & Testing
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: Automatic Trip Hammer

Depth in Feet	Depth in Meters	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
			SAND (SP), light yellowish brown, very loose, dry, fine-grained sand, <10% silt (yellowish brown, loose to medium dense, moist)										
	1					16							
5	2					10							
10	3		(wet, loose)		▽	5				8			
	4												
15	5					4							
			Bottom of boring at approximately 16 1/2 feet. Groundwater encountered at 11 feet during drilling.										

LOG OF BORING 1-B5

ENGEO INCORPORATED

Geotechnical Exploration
Solid Waste Transfer Station
Brentwood, California
8708.000.000

DATE DRILLED: 5/15/2009
HOLE DEPTH: Approx. 5 ft.
HOLE DIAMETER: 3.0 in.
SURF ELEV (msl): Approx. 60 ft.

LOGGED / REVIEWED BY: Z. Crawford / JJT
DRILLING CONTRACTOR: ENGEO Incorporated
DRILLING METHOD: Hand Auger
HAMMER TYPE: N/A

Depth in Feet	Depth in Meters	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
			SILTY SAND (SM), yellowish brown, loose to medium dense, dry to moist										
	1		(brown)										
5													
			Bottom of boring at approximately 5 feet.										
			Groundwater not encountered during drilling.										

LOG OF BORING 1-B6

ENGEO INCORPORATED

Geotechnical Exploration
Solid Waste Transfer Station
Brentwood, California
8708.000.000

DATE DRILLED: 5/15/2009
HOLE DEPTH: Approx. 5 ft.
HOLE DIAMETER: 3.0 in.
SURF ELEV (msl): Approx. 59 ft.

LOGGED / REVIEWED BY: Z. Crawford / JJT
DRILLING CONTRACTOR: ENGEO Incorporated
DRILLING METHOD: Hand Auger
HAMMER TYPE: N/A

Depth in Feet	Depth in Meters	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
			SILTY CLAY (CL), dark brown, stiff, dry to moist, <15% sand				45	20	25				
	1		SILTY SAND (SM), brown, medium dense, fine-grained sand										
5													

Bottom of boring at approximately 5 feet.

Groundwater not encountered during drilling.

ENGEO INCORPORATED

LOG OF BORING 1-B7

Geotechnical Exploration
Solid Waste Transfer Station
Brentwood, California
8708.000.000

DATE DRILLED: 5/15/2009
HOLE DEPTH: Approx. 5 ft.
HOLE DIAMETER: 3.0 in.
SURF ELEV (msl): Approx. 61 ft.

LOGGED / REVIEWED BY: Z. Crawford / JJT
DRILLING CONTRACTOR: ENGEO Incorporated
DRILLING METHOD: Hand Auger
HAMMER TYPE: N/A

Depth in Feet	Depth in Meters	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
			SILTY CLAY (CL), olive brown, stiff to very stiff, moist, <20% sand, roadway gravel at the surface										
	1	Hand sample	CLAYEY SAND (SC), dark brown, loose to medium dense, moist, fine-grained sand										
5													
			Bottom of boring at approximately 5 feet.										
			Groundwater not encountered during drilling.										

LOG OF BORING 1-B8

ENGEO INCORPORATED

Geotechnical Exploration
Solid Waste Transfer Station
Brentwood, California
8708.000.000

DATE DRILLED: 5/15/2009
HOLE DEPTH: Approx. 5 ft.
HOLE DIAMETER: 3.0 in.
SURF ELEV (msl): Approx. 46 ft.

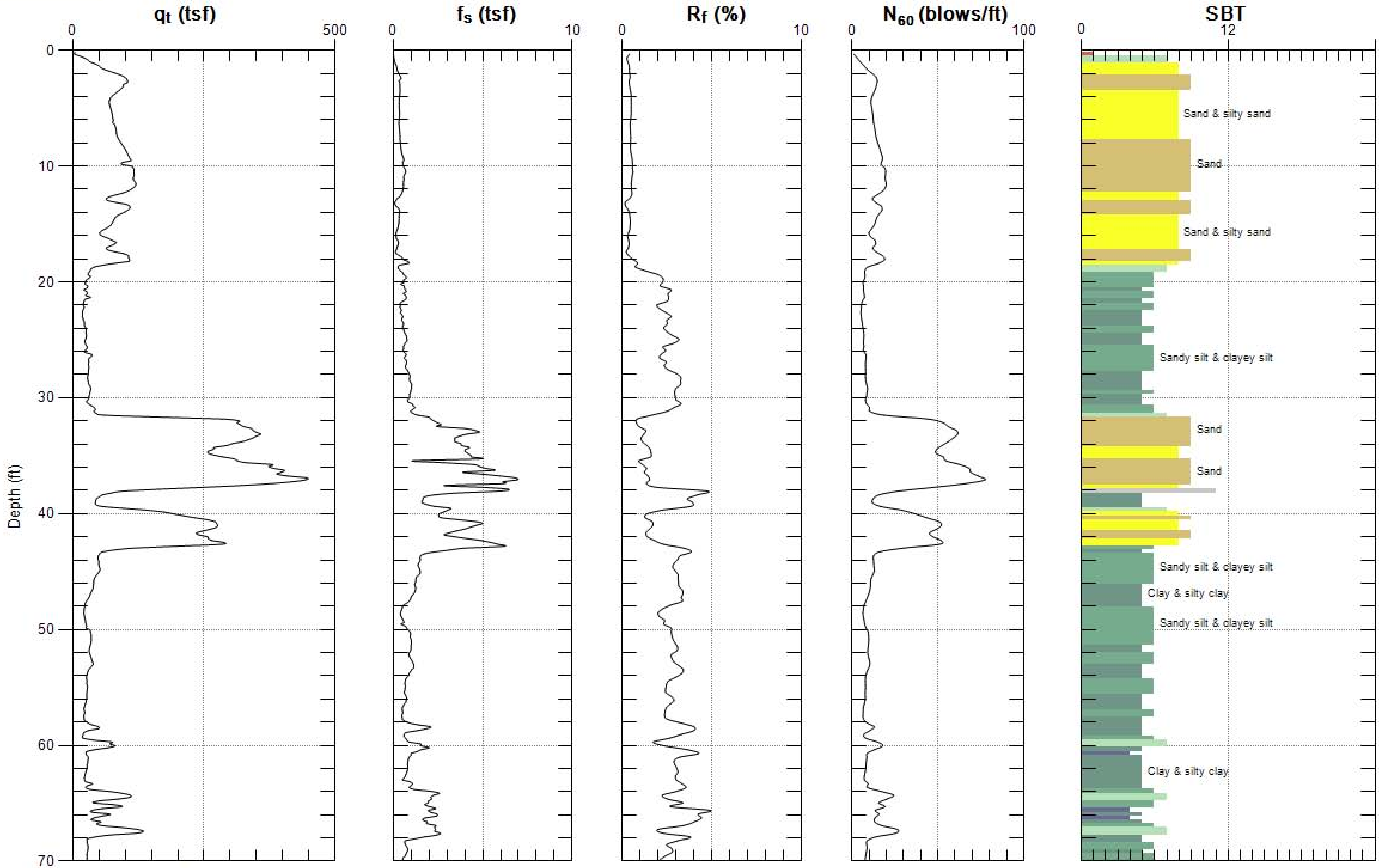
LOGGED / REVIEWED BY: Z. Crawford / JJT
DRILLING CONTRACTOR: ENGEO Incorporated
DRILLING METHOD: Hand Auger
HAMMER TYPE: N/A

Depth in Feet	Depth in Meters	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
			SANDY SILT (ML), dark yellowish brown, stiff to very stiff, dry to moist, fine-grained sand, <20% clay, trace fine gravel										
			CLAYEY SAND (SC), brown, medium dense, moist, fine-grained sand, <20% silt										
	1		SILTY SAND (SM), brown, medium dense, moist, fine-grained sand										
5													
			Bottom of boring at approximately 5 feet.										
			Groundwater not encountered during drilling.										

GREGG

ENGEO

Site: BRENTWOOD SWTS
Sounding: 1-CPT-01
Engineer: Z.CRAWFORD
Date: 5/15/2009 09:12



Max. Depth: 70.046 (ft)
Avg. Interval: 0.328 (ft)

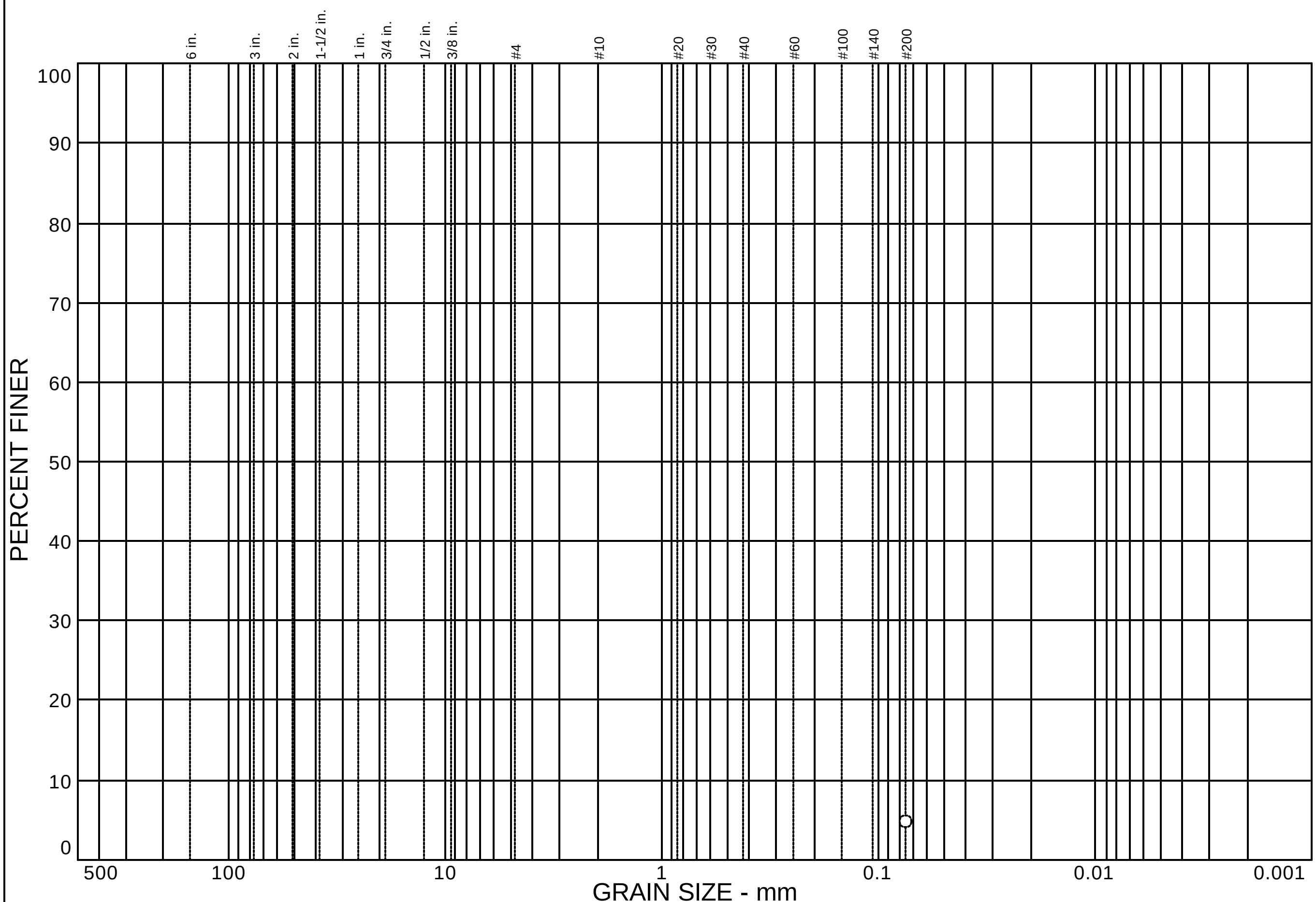
SBT: Soil Behavior Type (Robertson 1990)

APPENDIX B
LABORATORY TEST DATA

Particle Size Distribution Report (6 pages)
Liquid and Plastic Limits Test Report
R-Value
Evaluation for Soil Corrosion

APPENDIX B

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
			4.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	4.6		

* (no specification provided)

Soil Description

See Exploratory Boring Logs

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= D_{60}= D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= AASHTO=

Remarks

See Exploratory Boring Logs

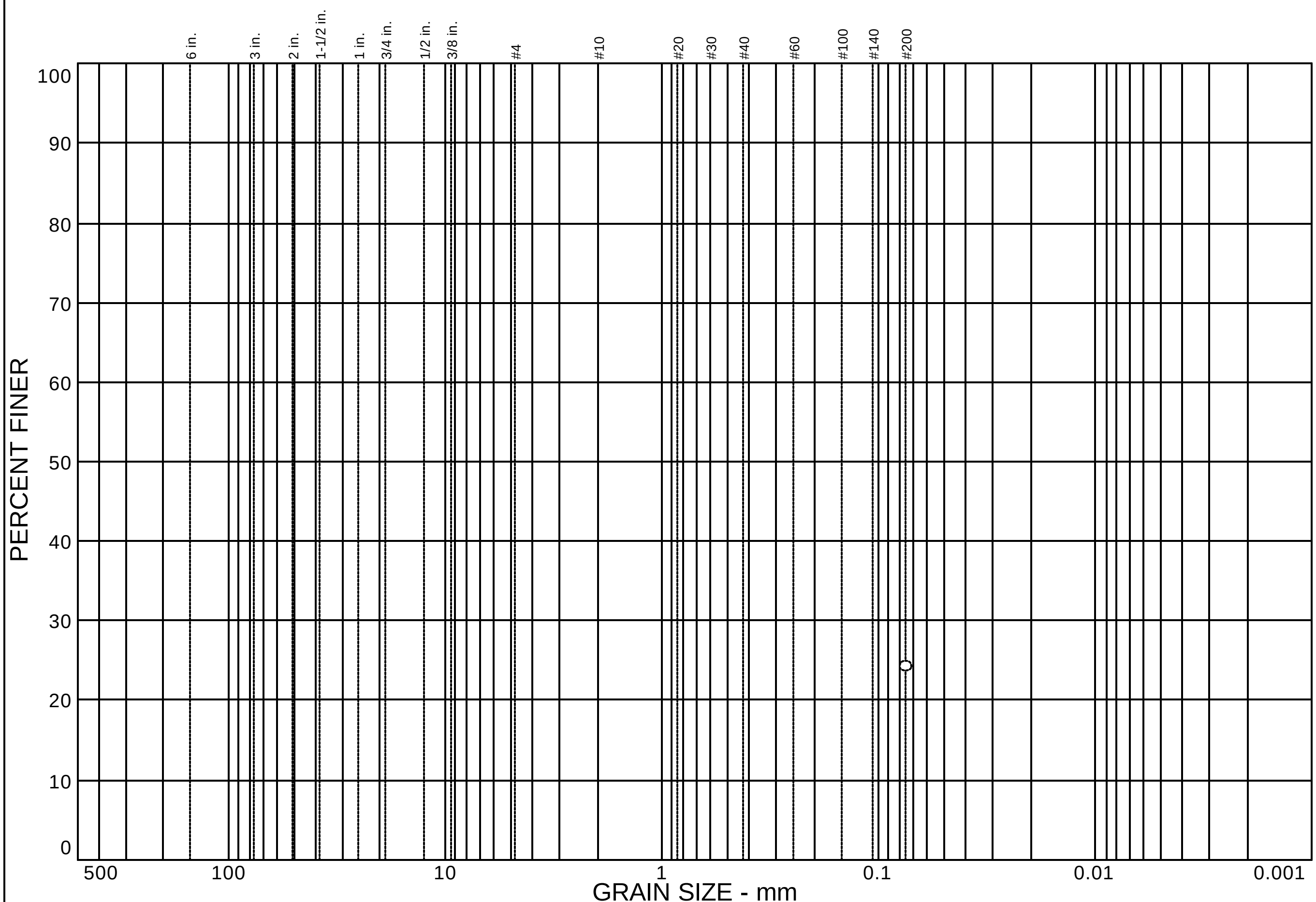
Sample No.: 1-B1 @ 2' **Source of Sample:** **Date:** 5-20-09
Location: **Elev./Depth:** 2'

ENGEO INCORPORATED
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS
MATERIALS TESTING

Client:
Project: Brentwood Transfer Station - Brentwood, CA

Project No: 8708.000.000 **Plate**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
			24.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	24.2		

* (no specification provided)

Soil Description

See Exploratory Boring Logs

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= D_{60}= D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= AASHTO=

Remarks

See Exploratory Boring Logs

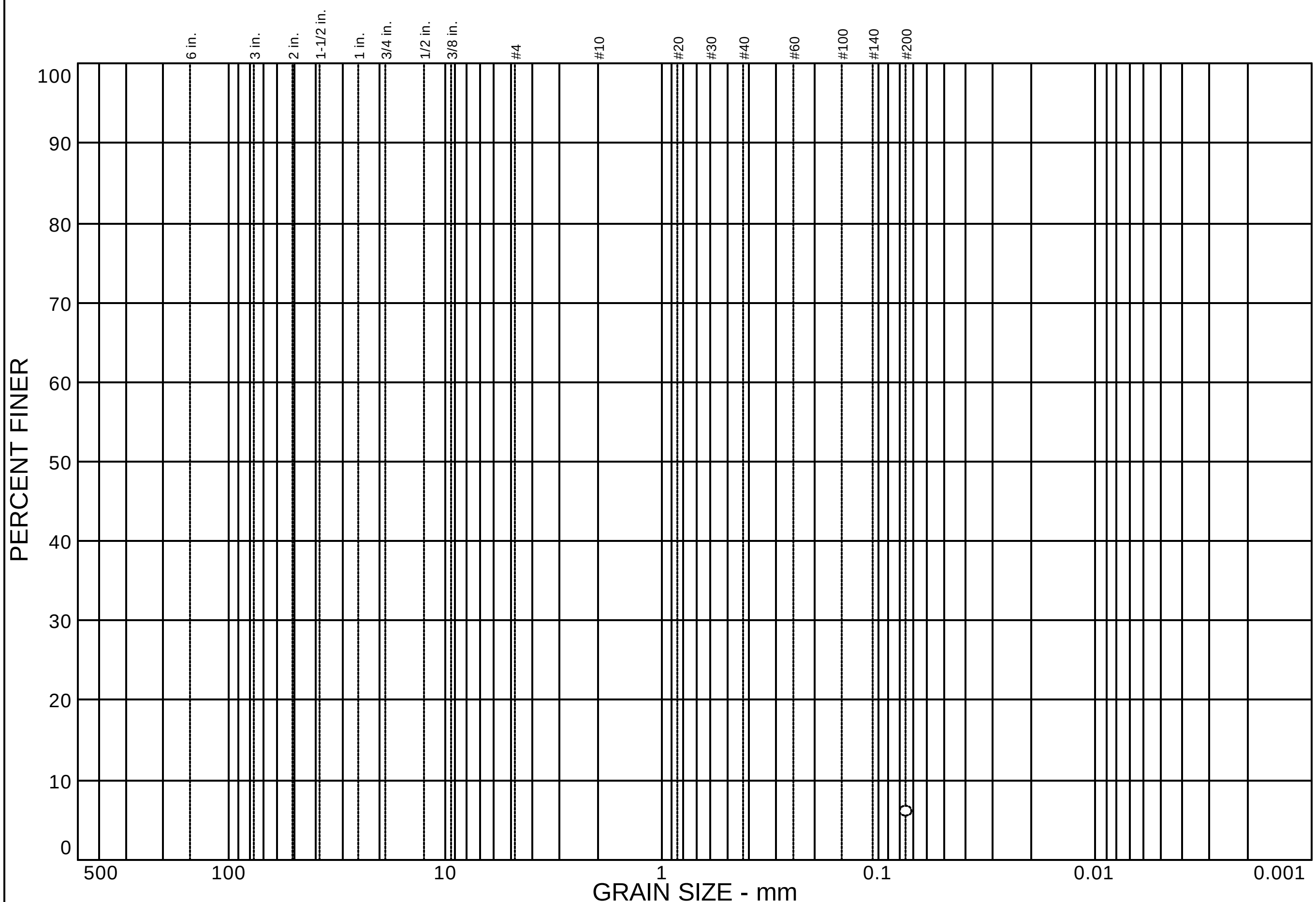
Sample No.: 1-B1 @ 16' **Source of Sample:** **Date:** 5-20-09
Location: **Elev./Depth:** 16'

ENGEO INCORPORATED
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS
MATERIALS TESTING

Client:
Project: Brentwood Transfer Station - Brentwood, CA

Project No: 8708.000.000 **Plate**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
			6.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	6.0		

* (no specification provided)

Soil Description

See Exploratory Boring Logs

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= D_{60}= D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= AASHTO=

Remarks

See Exploratory Boring Logs

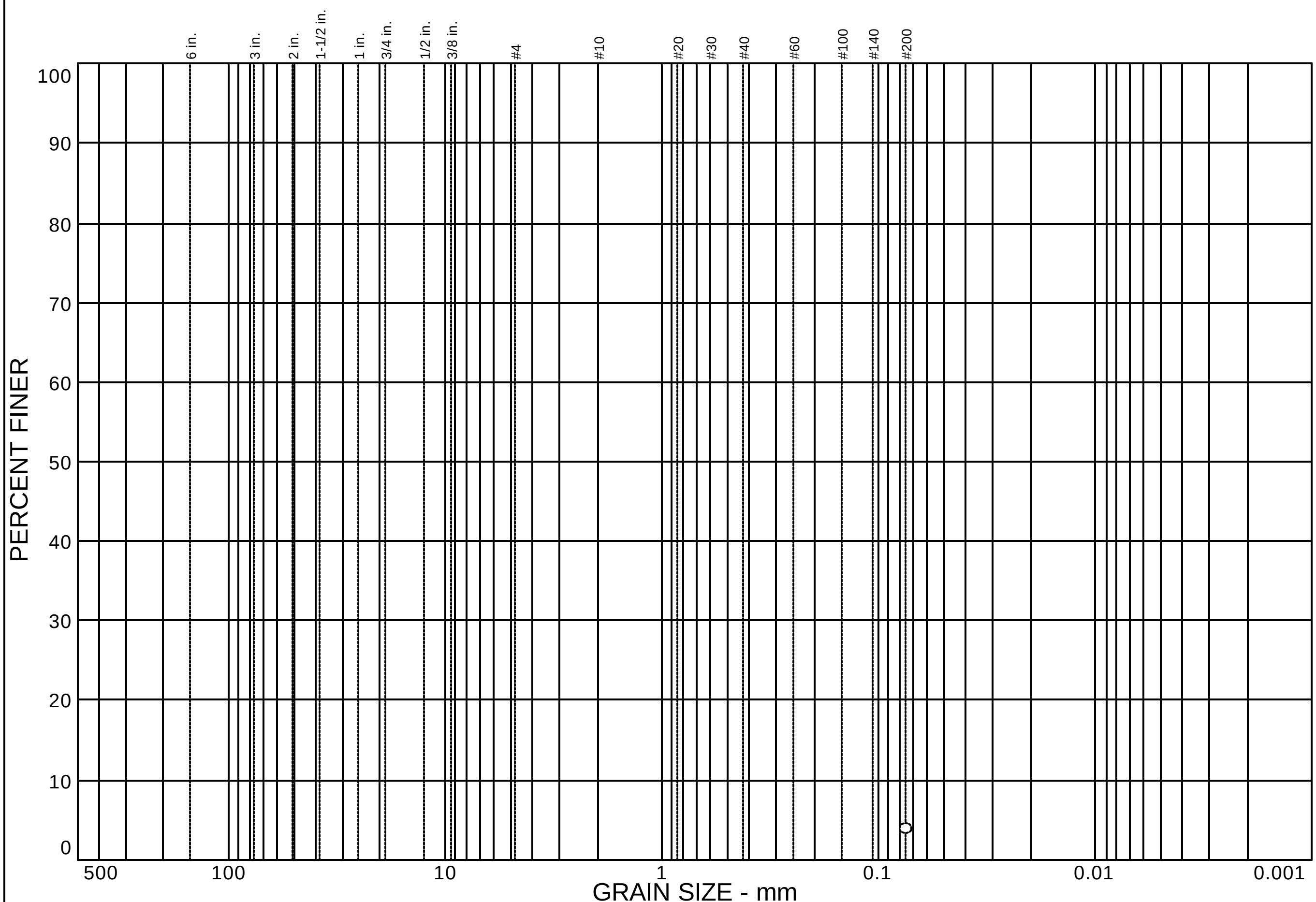
Sample No.: 1-B2 @ 16' **Source of Sample:** **Date:** 5-20-09
Location: **Elev./Depth:** 16'

ENGEO INCORPORATED — GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS MATERIALS TESTING

Client:
Project: Brentwood Transfer Station - Brentwood, CA

Project No: 8708.000.000 **Plate**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
			3.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	3.8		

* (no specification provided)

Soil Description

See Exploratory Boring Logs

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= D_{60}= D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= AASHTO=

Remarks

See Exploratory Boring Logs

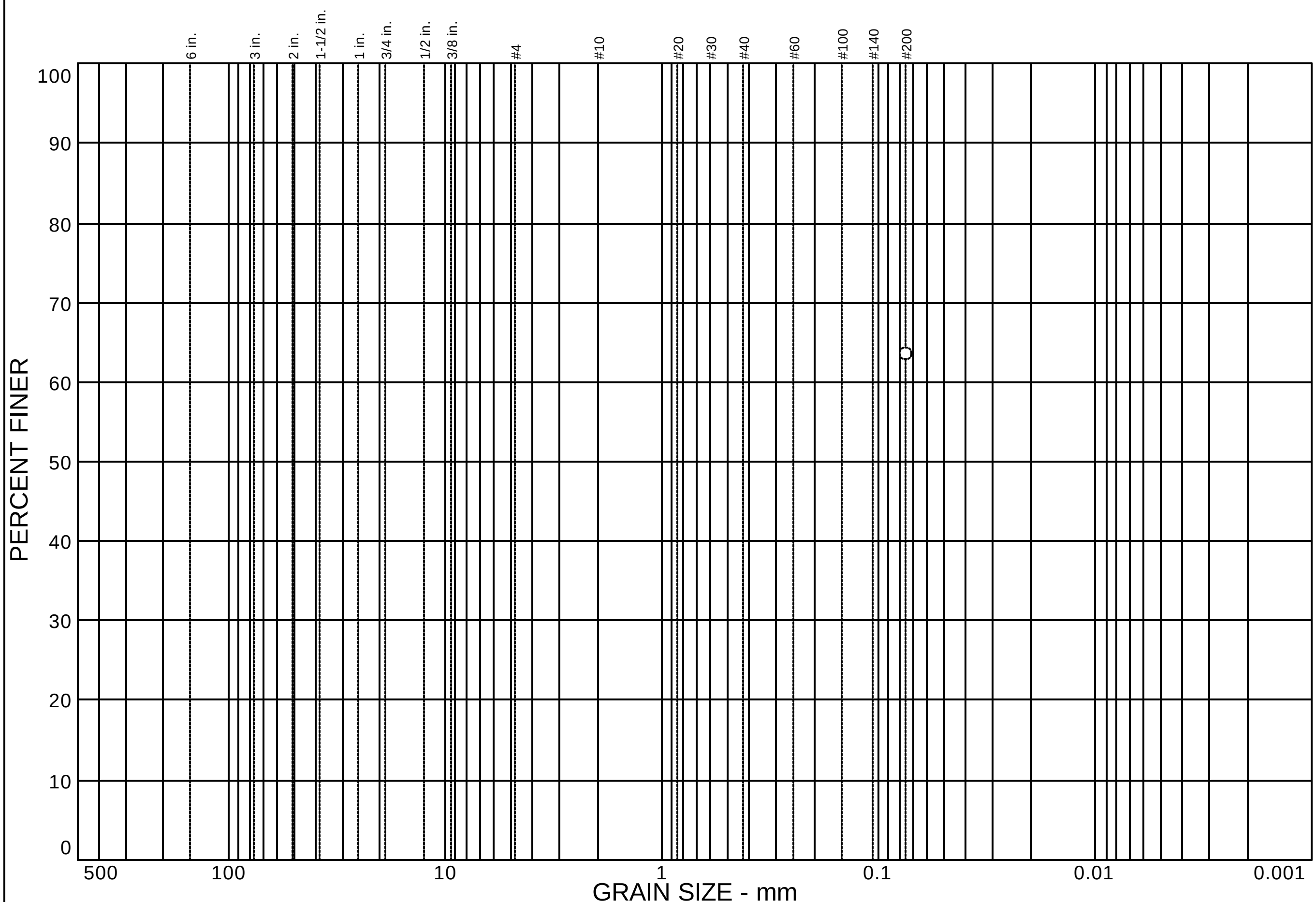
Sample No.: 1-B3 @ 16' **Source of Sample:** **Date:** 5-20-09
Location: **Elev./Depth:** 16'

ENGEO INCORPORATED
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS
MATERIALS TESTING

Client:
Project: Brentwood Transfer Station - Brentwood, CA

Project No: 8708.000.000 **Plate**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
			63.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	63.5		

* (no specification provided)

Soil Description

See Exploratory Boring Logs

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= D_{60}= D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= AASHTO=

Remarks

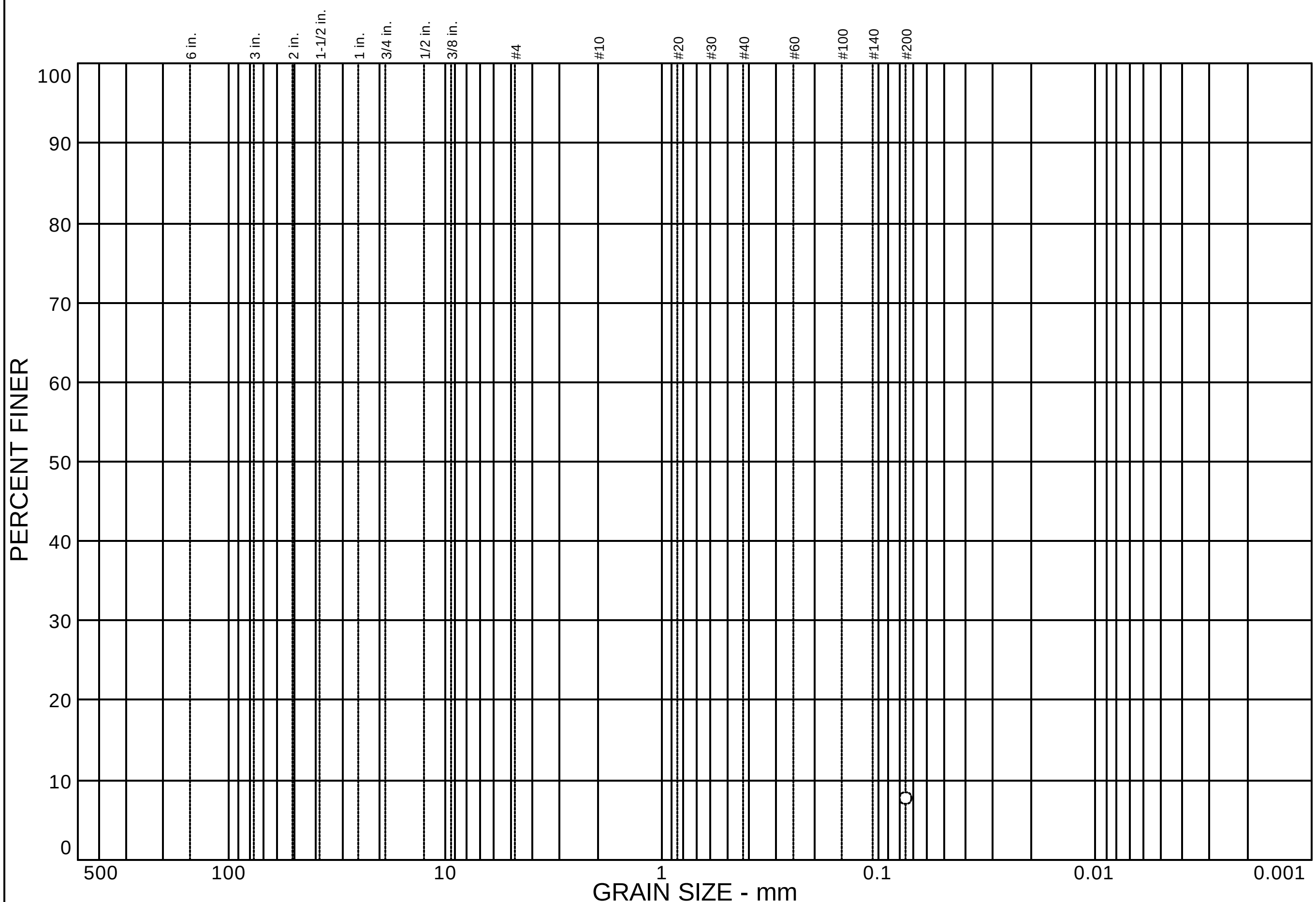
See Exploratory Boring Logs

Sample No.: 1-B3 @ 21' **Source of Sample:** **Date:** 5-20-09
Location: **Elev./Depth:** 21'

ENGEO INCORPORATED
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS
MATERIALS TESTING

Client:
Project: Brentwood Transfer Station - Brentwood, CA
Project No: 8708.000.000 **Plate**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
			7.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	7.6		

* (no specification provided)

Soil Description

See Exploratory Boring Logs

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= D_{60}= D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= AASHTO=

Remarks

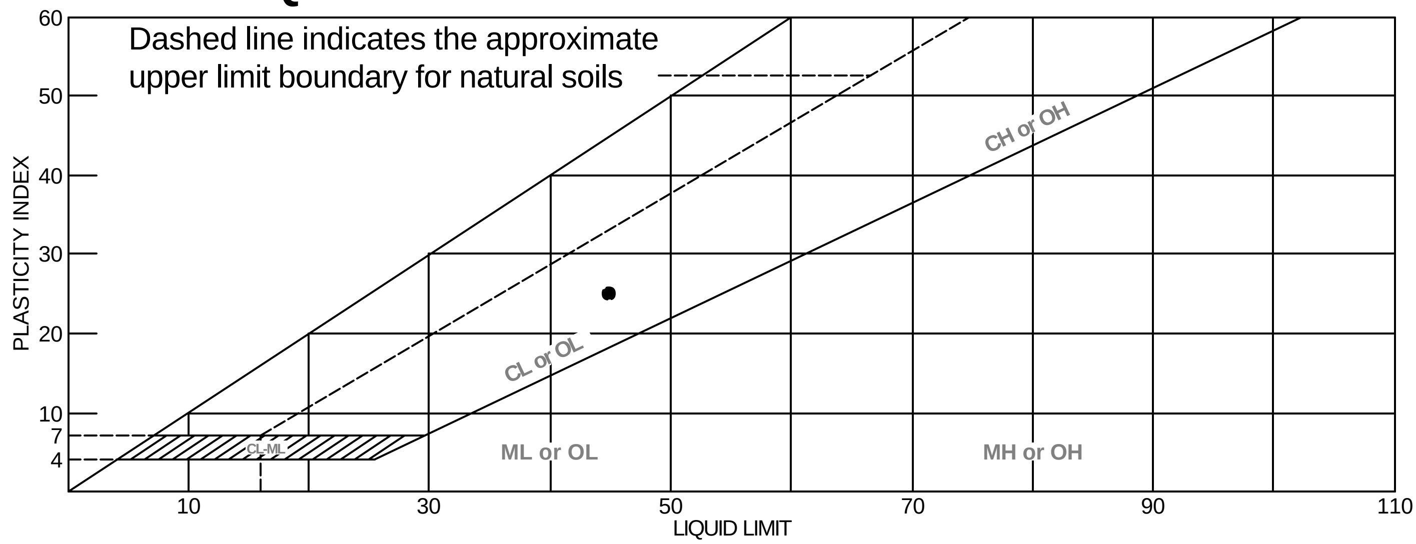
See Exploratory Boring Logs

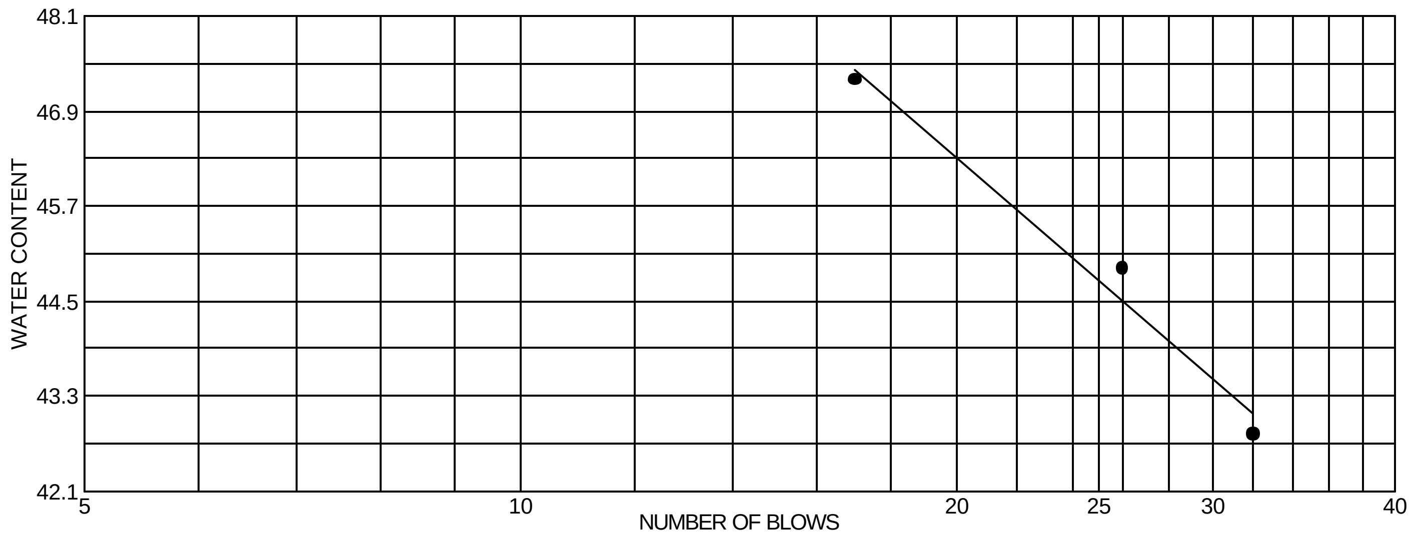
Sample No.: 1-B4 @ 11' **Source of Sample:** **Date:** 5-20-09
Location: **Elev./Depth:** 11'

ENGEO INCORPORATED
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS
MATERIALS TESTING

Client:
Project: Brentwood Transfer Station - Brentwood, CA
Project No: 8708.000.000 **Plate**

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	See Exploratory Boring Logs	45	20	25			

Project No. 8708.000.000 **Client:**

Project: Brentwood Transfer Station - Brentwood, CA

● **Source:** **Sample No.:** 1-B6 @ 1' **Elev./Depth:** 1'

ENGEO INCORPORATED GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS MATERIALS TESTING

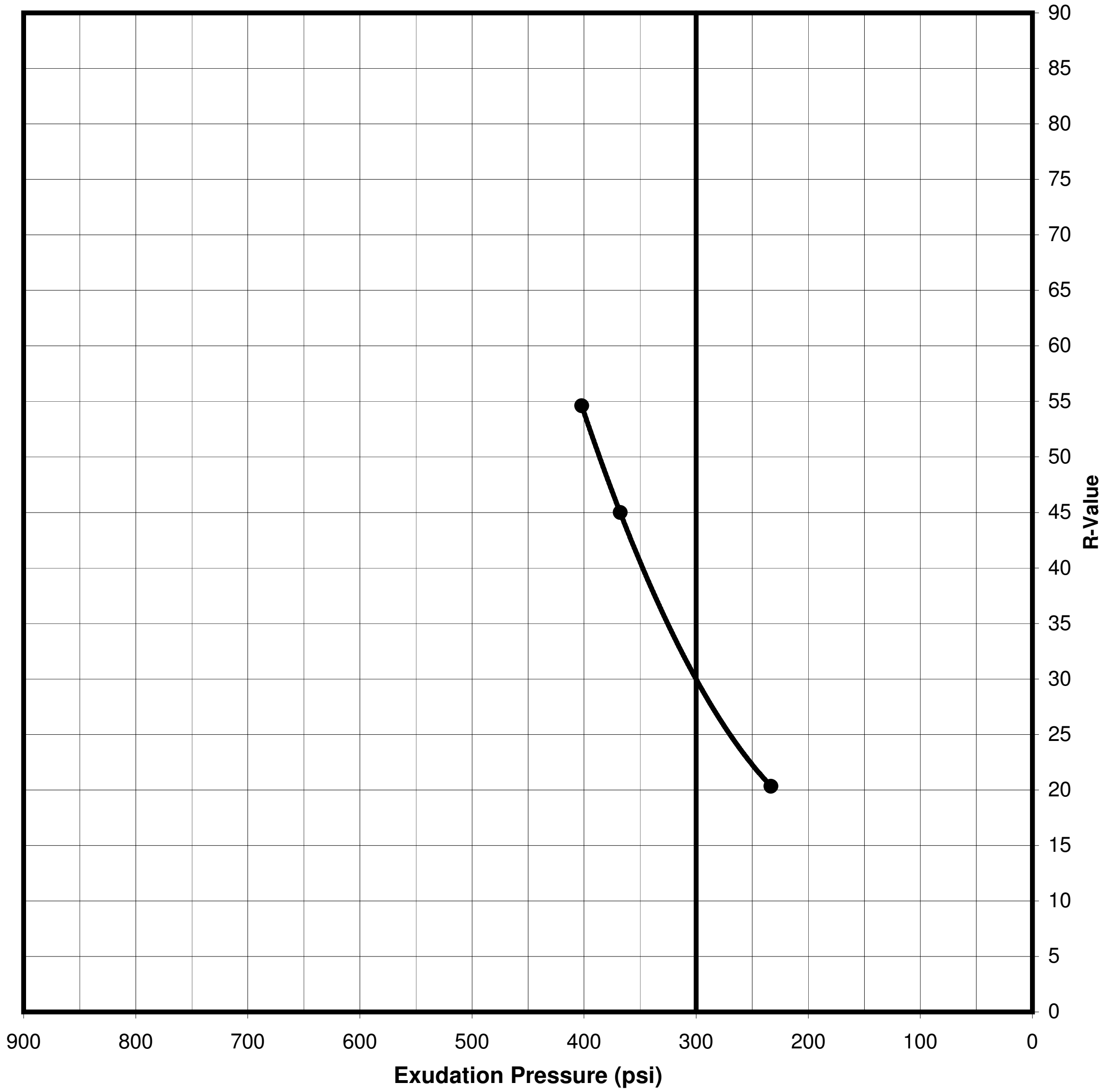
Remarks:

● See Exploratory Boring Logs

Plate

ENGEO INCORPORATED

R VALUE TEST REPORT
CAL-301



Date: 5/21/09
Project Name: Brentwood Solid Waste Transfer Station
Project Number: 8708.000.000
Sample: Rv 1
Description: Dark yellowish brown sandy SILT

Specimen	A	B	C
Exudation Pressure, p.s.i.	402	368	233
Expansion dial (.0001")	51	15	0
Expansion Pressure, p.s.f.	221	65	0
Resistance Value, "R"	55	45	20
% Moisture at Test	10.5	12.2	14.0
Dry Density at Test, p.c.f.	123.6	120.7	117.8
"R" Value at 300 p.s.i., Exudation Pressure	30		

Sunland Analytical

11353 Pyrites Way, Suite 4
Rancho Cordova, CA 95670
(916) 852-8557

Date Reported 06/03/2009
Date Submitted 05/28/2009

To: Zac Crawford
Engeo Inc.
580 No. Wilma Ave, Suite A
Ripon, CA 95366

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : 8708/BRENTWOOD TRANS Site ID : B1 @ 1.5'.
Thank you for your business.

* For future reference to this analysis please use SUN # 55778-112476.

EVALUATION FOR SOIL CORROSION

Soil pH	4.41		
Minimum Resistivity	2.28	ohm-cm (x1000)	
Chloride	14.5 ppm	00.00145	%
Sulfate	267.6 ppm	00.02676	%

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422